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Tests on a Full-Scale Rigid Jointed Multi-Storey Steel Frame

Tests à échelle réelle d'un câdre en acier de plusieurs étages

Prüfung eines maßstäblichen, steifknotigen Stockwerkrahmens

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Introduction

The use of rigid jointed multi-storey steel frames, and the achievement of the economies which undoubtedly would accrue, has been inhibited for two reasons. Firstly, the difficulty of achieving in practice the necessary degree of continuity at the joints and secondly the complexity of design. With the development of site welding and its feasibility for effecting truly rigid joints (together with the development of the High Strength Friction Grip bolt) the former problem has been largely overcome. The latter problem, the evolution of a practical design method, has been the subject of study by a committee, set up jointly by the Institute of Welding and the Institution of Structural Engineers in 1957. Their first Report was published in December 1964. Rigorous analysis, taking into account full continuity and all possible loading combinations, remains intractable. However, by making certain simplifying assumptions, and by presenting in tabular form the results of a computer programme, this Report presents a logical and practical method of design for such frames. This contribution describes briefly a series of tests on a full scale frame carried out to verify the method and to determine the degree of error inherent in the simplifying assumptions.

Summary of Design Principles

The basic principle of the Report method is that of ultimate load design and being more accurate than simple design hitherto common it proposes that an overall load factor of 1.5 can safely be adopted, (instead of the usual 1.75).

The fundamental assumptions are as follows:-

(a) Steel is mild steel to British Standard 15, with a yield stress of 16.0 ton f/in^2 (25.2 kgf/mm^2).

(b) Connections between beams and columns have the full rigidity that can be made possible by welding. (The Report applies also to frames in which full rigidity of connections can be provided by HSFG bolts).

(c) The beams receive no assistance from composite action with the floors they support, and the columns no assistance from encasement.

(d) Lateral forces are resisted separately by walls or bracing, the steel frame being thus relieved of sway.

Individual beams and columns are designed on the following principles:-

(i) Major-Axis beams, i.e. beams which at both ends bend or restrain the columns about their major axes, are designed in accordance with the plastic theory. In general, a major-axis beam is designed on the assumption that the beam is "fixed-ended", developing under factored loading three plastic hinges, one at each end and one in the middle. The assumption of "fixed ends" is justified provided that at each end of the beam the sustaining moment supplied by the adjoining beam and columns is at least equal to the hinge moment at that end. Where this condition is not satisfied, the magnitude of the moment at the end of the beam reduces to that of the sustaining moment.

(ii) Minor-Axis beams, i.e. those other than major-axis beams, are designed elastically for factored loading to a limiting extreme fibre stress of 16.0 tonf/in^2, in order to provide elastic restraint to the columns about their minor axes. Here the stress in the beam is dependent on the loading and stiffness of all other members of the frame, and hence exact assessment is most complex. To overcome this difficulty the report introduces the "limited-frame" concept. This assumes that it is sufficiently accurate to consider only that limited part of the frame to which the member is connected (in the plane of bending). Further, that the remote ends of this limited frame can be considered as fixed. (See Fig. 1)

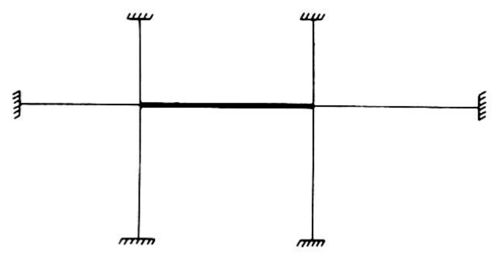

FIG 1 THE LIMITED FRAME FOR BEAM DESIGN

Having established the critical combination of dead and live loading applied to the limited frame, the support moments at the ends of the beam are found by a moment distribution procedure.

(iii) Columns

Evaluation of bending moments entails the use of the limited-frame concept in a manner similar to that employed in the design of minor-axis beams, except that in this case two limited-frames need to be considered, one in the plane of the major-axis and one in the plane of the minor-axis (see Fig. 2). Axial and bending stresses can then be found.

To these stresses are added firstly those due to axial load acting on the initially curved columns, and secondly those extra stresses arising from the axial load operating on the column as bent by the beams.

These incremental stresses have been evaluated on a computer for a wide range of values of axial stress, joint stiffness ratios, type of curvature and slenderness ratio. The results are presented in the Report in the form of design tables of reduced permissible column stresses (i.e. yield stress less the incremental stresses).

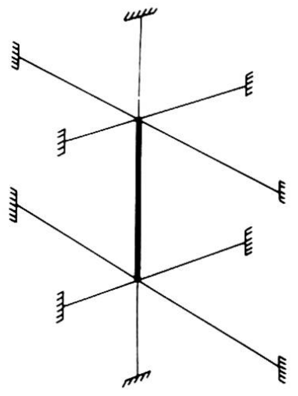

FIG. 2 THE LIMITED FRAME FOR COLUMN DESIGN

The design criterion adopted is that of attainment of yield stress at the extreme fibres in single curvature bending under factored load. This recognises that some plasticity will occur at the ends of a column under double curvature bending but can be safely accommodated.

Experimental Procedure

The experimental work was carried out through a collaborative agreement between the British Iron & Steel Research Association and the Government Building Research Station, whereby BISRA designed and secured a frame and BRS provided laboratory accommodation and services. Testing to a jointly agreed programme was carried out by personnel from both organisations.

Details of Test Frame

In order fully to verify the Report a 5-storey 5 x 5 bay frame would be needed. Consideration was given to testing a model but was rejected on various grounds.

Having regard to the available laboratory space, a 3 storey 2 x 1 bay frame was designed having the form shown in Fig. 3. The frame was designed to sustain at failure the following loadings:-

	Roof Level	2nd Floor Level	1st Floor Level
Dead Load	50 lb/Ft^2 (244 kg/m^2)	75 lb/Ft^2 (366 kg/m^2)	75 lb/Ft^2 (366 kg/m^2)
Live Load	80 lb/Ft^2 (391 kg/m^2)	90 lb/Ft^2 (439 kg/m^2)	90 lb/Ft^2 (439 kg/m^2)

It was assumed that one way spanning floor slabs, with intermediate beams, would be carried as uniformly distributed load for minor-axis beams and centre point loads for major-axis beams. For the test, point loads at quarter-points represented U.D.L. The range of available sections is such that many members were over-designed.

Sway was prevented by tying the frame to the laboratory balconies. Rigid joints were effected with High Strength Friction Grip bolts. Intermediate beams were made heavy so as to exclude premature yielding in them.

The average yield stress of the steel was found to be 19 tonf/in^2 (30 kgf/mm^2). This was not taken into account in design.

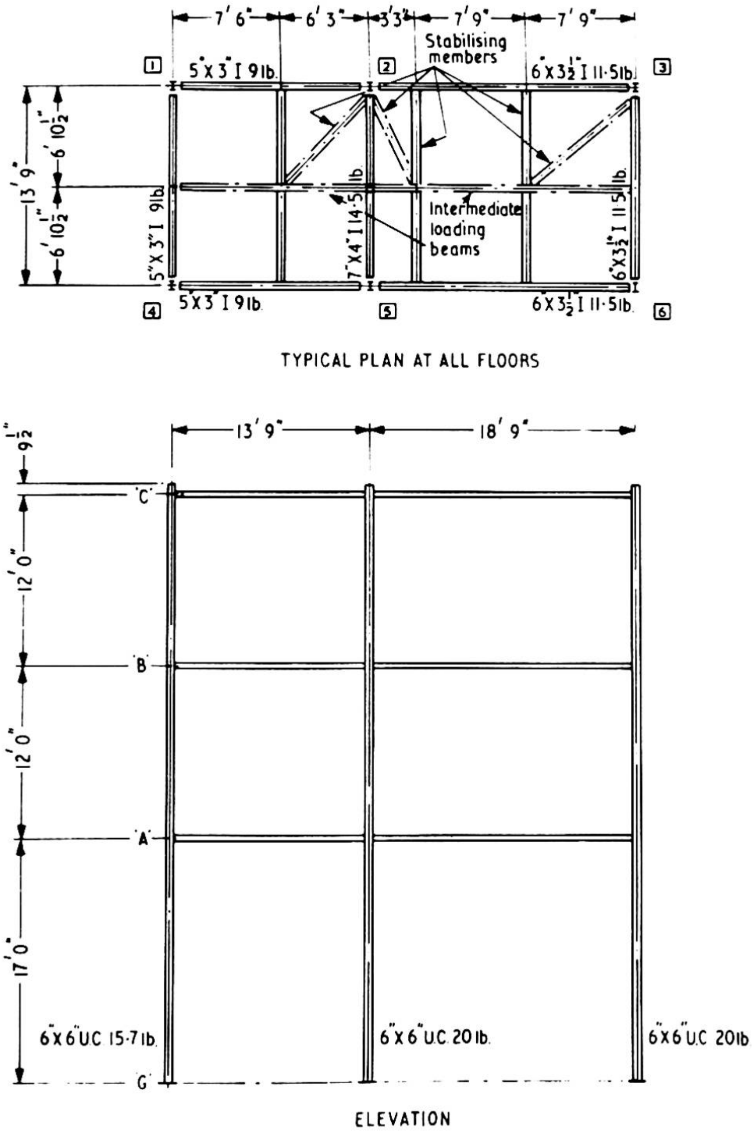

FIG. 3 OVERALL FRAME DIMENSIONS

Loading and Instrumentation

Loading was applied to beams and columns by means of spreader beams acted upon by cables passing through the laboratory strong floor. These were tensioned by hydraulic jacks reacting against the underside of the floor. A total of 396 electrical resistance strain gauges were fixed to the structure in groups of four, at the beams and columns at the joints, and at the mid-span of the beams. Transducers measured deflections of beams at mid-span and of columns at five points about both axes. Load cells measured cable forces.

Test Programme

In an ideal test frame, at the factored working load major-axis beams should have developed three plastic hinges, and minor-axis beams and columns should have attained yield at the extreme fibres To achieve this the following conditions should obtain:-

(i) A wholly accurate design method.

(ii) The ability to provide sections with exactly the right properties.

(iii) An accurate knowledge of the yield stress.

These conditions cannot be achieved in practice and it was item (i) which was being investigated in this case.

Testing was therefore carried out in the following stages:

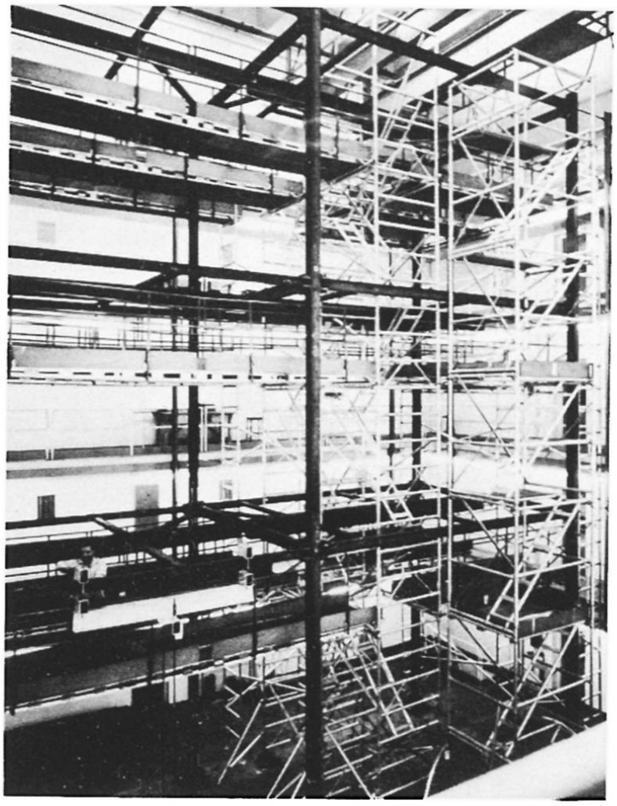

Fig. 4 - Overall View of Test Frame

Stage 1

Stanchions and beams were loaded in turn to their design loads (i.e. working load x 1.5).

Stage 2

Stanchions and beams were loaded up to their limiting values, as calculated by the Report method, taking discrepancies due to (ii) and (iii) above into account.

Stage 3

Test loads were successively increased until strain and deflection measurements confirmed that limiting conditions, as defined by the Report, had been attained. (Thus the difference between Stage 2 and Stage 3 would indicate the degree of conservatism in the design method).

Stage 4

Loading was continued to outright collapse of columns, by imposing additional axial load (major-axis beams having already attained full plasticity.)

Summary of Results

Stage 1

As was to be expected all frame members remained elastic throughout.

Stage 2

Major-axis beams achieved central plastic hinges, but although some plasticity was achieved at the supports, full plastic hinges were not developed there.

Adjacent to the centre columns, minor-axis beams reached yield stress.

Peak stresses in the columns were in all cases less than the yield stress.

Stage 3

Major-axis beams reached full plasticity at three points.

Minor-axis beams yielded.

Columns were loaded so as to produce the maximum single curvature bending condition and axial load was applied to produce yielding.

In general the loads applied to a group of members under test were increased until one or other of these members reached its limiting condition. When that occurred the load on that member was maintained whilst the loads on the remaining members were increased until the next failure, and so on. Thus at this stage loading departed from a hypothetical floor loading.

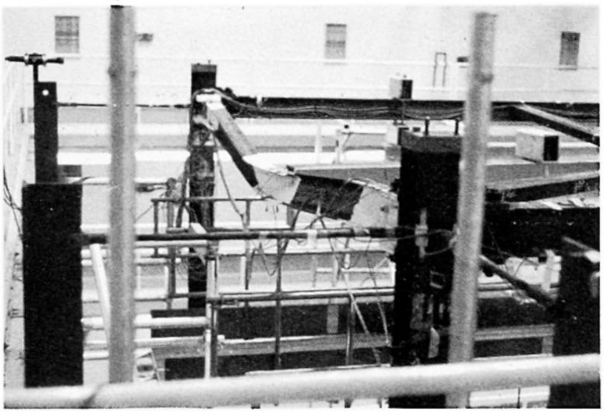

Fig. 5 – Major-Axis Beam at Collapse

Stage 4

Three column lengths were loaded to collapse, one at each floor level. They were first subjected to the same beam end moments as in Stage 3 and then direct axial load was applied until collapse.

Conclusions

Having in Stage 2 applied loads which compensate for the chosen sections and for a yield stress of 19 $tonf/in^2$ (30 kg f/mm^2), it can be asserted that the design method is safe.

From the results of Stage 3 loading the following broad conclusions can be drawn:-

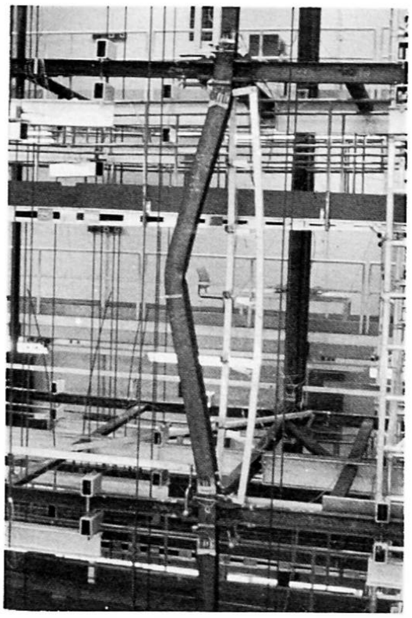

Fig. 6 - Column at Collapse

(i) Major-axis beam design was slightly conservative, the ultimate loads being underestimated.

(ii) The minor-axis beams were slightly underdesigned in that the limited frame concept gave slightly smaller design moments than an accurate elastic analysis.

(iii) The measured column stresses were consistently slightly less than the stresses predicted by the Report. It was clear from Stage 4 loading that a considerable margin for the addition of axial load exists beyond the limiting condition as defined by the Report, i.e. the attainment of yield in the extreme fibres under single curvature bending. These results show that the criterion of column design could be considerably improved by subsequent research leading to an accurate criterion for collapse with increased plasticity.

In short, the Joint Committee Report presents a logical design method for rigid jointed frames, which is reasonably accurate and which can be applied with confidence.

Future Work

Following the publication of the first Report, the Joint Committee has been reconvened to extend the method to include such things as the use of High Tensile Steel, composite action of beams, concrete encased columns, plastic design of minor-axis beams, etc.

At the time of writing it is hoped that a second frame will be tested by the same procedure. It will be of larger dimensions, incorporating an internal column, and will be of High Tensile Steel to BS.968.

Acknowledgements

This work is more fully reported in a paper presented to the Institution of Structural Engineers in April, 1968 by R.H. Wood, R.F. Smith and the present author, who would like here to pay tribute to his colleague's work.

This project formed part of the respective programmes of the Building Research Station and BISRA and is published by permission of the Directors.

SUMMARY

This contribution briefly describes a new design method, based on certain simplifying assumptions, for treating rigid-jointed multi-storey frames, and which was published in 1964. By a collaborative arrangement a 2 bay x 1 bay frame, 3 stories high was designed by BISRA to this method and tested at the Building Research Station Laboratory. A series of tests were carried out at working load and to failure, to verify the design method and to establish actual load factors.

RÉSUMÉ

Ceci est la brève description d'une nouvelle méthode d'études, basée sur certaines hypothèses simplificatrices, du traitement des câdres à plusieurs étages, publiée en 1964.

Un câdre de trois étages, consistant en un arrangement de 2 x 1 câdre fut conçue selon cette méthode par BISRA et testée au Building Research Laboratory. Une série de tests fut menée dans les conditions de travail et jusqu'au point de rupture, pour vérifier la méthode d'étude et pour établir les facteurs de charge réelle.

ZUSAMMENFASSUNG

Dieser Beitrag beschreibt in knappen Worten eine neue Bemessungsregel für steifknotige Stockwerkrahmen, die auf einigen vereinfachenden Annahmen beruht und 1964 veröffentlicht worden war. Ein 2mal eine Abteilung umfassender, dreistöckiger Rahmen ist von der BISRA entworfen und beim Building Research Station Laboratory geprüft worden. Eine Versuchsreihe ist für Arbeits- und Traglast durchgeführt worden, um die plastische Bemessung zu erhärten und Nutzlastwerte zu finden.